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STRUCTURAL CHARACTERISTICS OF BRICK MASONRY

BY HOWARD R. STALEY

(Presented at a meeting of the Designers' Section of the Boston Society of Civil Engineers
held on April 10, 1940)

IN THE consideration of the structural use of any material or combination of materials the first thought is usually that of strength and while this property is generally the controlling factor in design, other factors are commonly considered. In the case of masonry, the ability of the combination of brick and mortar to exclude the elements is certainly an important function of the wall. In many cases, where the walls are merely curtain-walls, the exclusion of the elements is practically their only function. Structural integrity is directly related to the ability of the wall to exclude the elements in any instance.

Generally speaking many of the factors used in structural design are the result of experience in use. Unless a material or combination of materials is being used for the first time it is only necessary to examine the experience record of the materials contemplated for use in order to arrive at a logical and safe conclusion. One can learn much about the structural value of any material and its ability to perform the functions for which it was chosen merely by an acquaintance with its behavior in use in the field when subjected to the various forces. Obviously, the strength at 28 days is not a single criterion of structural value even for a homogeneous material and this value becomes of even more questionable importance when considering a heterogeneous material, like masonry. Perhaps the above facts are not so obvious to those who are still imbued with the

strength-complex, but even to the obstinate it is becoming increasingly evident that possibly durability and structural integrity are factors for consideration. It might be well to emphasize the fact that strength and durability are not synonymous. As to durability there are thousands of examples of durable masonry around us. Similar combinations can be expected to give equivalent results today.

It is proposed in this paper to discuss some of the significant factors that have a bearing upon the selection and use of materials for brick masonry. To discuss all of the various factors involved would make the paper of prohibitive length, so it has been limited to include those factors which are important from a structural stand-

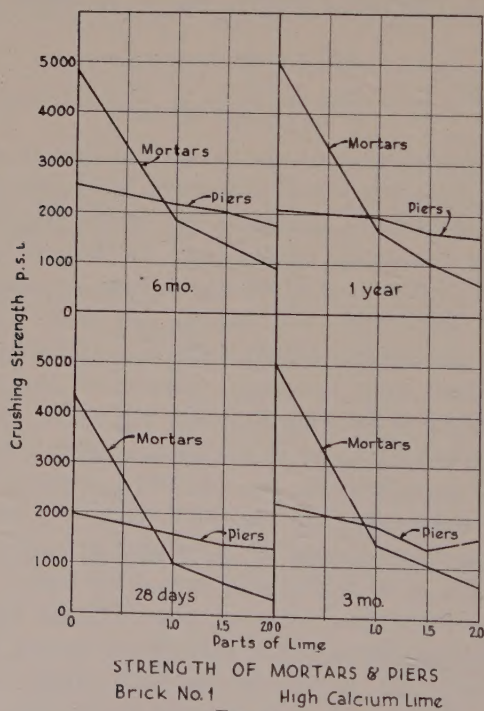
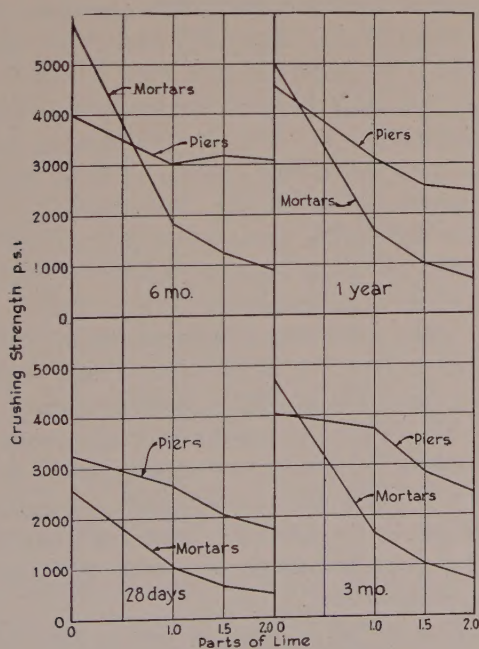


FIG. 1

point. These include strength and the various related properties, volume changes and their effect, and a consideration of stress conditions due to various factors. While not necessarily of greatest importance, strength will be considered first.

The series of pier tests reported were planned to investigate the effect upon masonry strength of seven mortars of four proportions, when assembled with five representative brick. Since it has long been known that the strength of mortar and brick, when tested separately, do not give a clear indication of the strength of the combination, masonry piers were used for the tests. The mortars used in this series of tests were made from a normal portland cement, combined with both a dolomitic and high-calcium quicklime putty and sand with a fineness modulus of 1.72. All lime putty was soaked 72 hours after slaking before mixing. The mortars were machine-mixed and were made to a consistency suitable to the mason who built the piers. The proportions given for the mortars are by volume of the dry materials, cement, lime-hydrate and sand. Brick Nos. 1 and 4 are typical back-up brick, while the others would be classified as face-brick.

In building the piers, the brick were laid dry, all joints were



STRENGTH OF MORTARS & PIERS
Brick No. 2 High Calcium Lime
FIG. 2

slushed full and the face of the joints were struck with a flat jointer. All specimens were moist-cured for 28 days and remained in the laboratory air for the balance of the period until tested.

The pier tests were made on 8" x 8" x 16" (6 courses high) piers laid up with $\frac{1}{2}$ " joints. Parallel tests were made upon 2" x 4" mortar cylinders and "two-bat" assemblages of which the latter are not reported here. The values given are the average of two piers and three 2" x 4" cylinders. During the loading of the piers strain-gage readings were taken at 10,000 pound increments of load up to 160,000 pounds and thereafter at 20,000 pound increments. The results of the tests together with other significant data are recorded in Table 1.

STRENGTH OF PIERS

In Figs. 1-5 the crushing strengths of the piers and mortars made with high-calcium lime are plotted against parts of lime-hydrate. These plots are typical of the piers and mortars made with the dolomitic lime, except that both strength values are quite generally higher with the use of the dolomitic lime. Of particular interest as far as load-bearing capacity is concerned are Figs. 1 and 4 as Brick Nos. 1 and 4 are certainly more typical of the back-up brick which usually take the major portion of the loads in wall-bearing structures. It is quite evident that with Brick No. 1 (Fig. 1) little is gained from the standpoint of strength by using the high-strength portland cement mortar rather than that using one part of lime. Tests made by Dr. N. Davey in England,* using brick of a lower compressive strength than used in the series reported here, show a similar relation to that shown in Fig. 1. The foregoing statement about strength for Brick No. 1 is not quite as true for Brick Nos. 2 and 4 (Figs. 2 and 4) but even in these cases the strengths are adequate. A study of all these plots shows the fallacy of requiring or accepting mortar composition as an absolute indication of qualification for load-bearing capacity. Allowable loads for masonry are sometimes based solely upon the composition of the mortar. Such requirements show a misunderstanding of the qualifications of mortar necessary for masonry strength as well as durability.

With Brick Nos. 3 and 5 (Figs. 3 and 5) the pier strengths are

*Clay Products Technical Bureau of Great Britain, Bulletin No. 6, Mortar Composition and Its Influence on the Compressive Strength of Brickwork. Dr. N. Davey.

higher than the mortar strengths at all ages except with Brick No. 5 at one year, when the pier and mortar strengths are approximately equal. It should be noted that the use of Brick No. 3 which has a compressive strength of 10,400 pounds per square inch as compared to 9500 pounds per square inch for Brick No. 5 results in lower pier strengths in all instances of equivalent comparison. This can be accounted for in two ways. Brick No. 3 has a lower strength in modulus of rupture (See Table 1) than Brick No. 5 and No. 3 is much more irregular in shape than No. 5. Due to the irregularity in shape of Brick No. 3, the mortar joints in masonry built of this brick

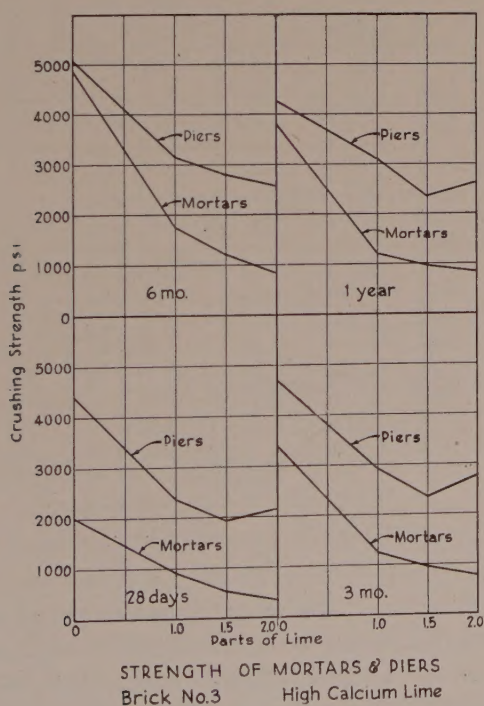


FIG. 3

are of variable thickness, thus the loads on such masonry are somewhat concentrated at the thinner portion of the joint because of a lesser resistance to deformation of the mortar in the thicker part of the joint. The result of this unequal pressure subjects the brick to

bending and failure occurs at a smaller load than if the brick were straight and of uniform size. Initial failure of masonry piers usually occurs because of failure of the brick-headers in modulus of rupture. Complete failure is caused by column action in the withe which has been released from the lateral restraint of the headers.

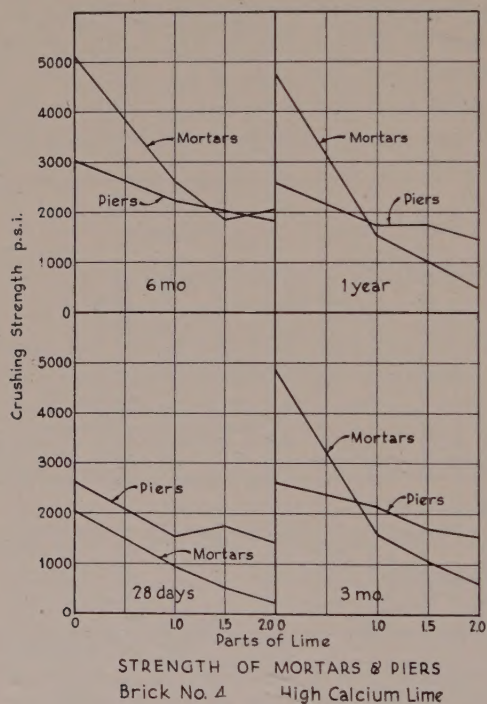


FIG. 4

In this series of tests, for the clay brick alone, the strength of the piers followed the strength of the brick in modulus of rupture more closely than the compressive strength of the brick. This may or may not be a significant factor, as the strength of brick in modulus of rupture usually varies approximately as the compressive strength.

In all instances the use of portland-cement mortar and Brick No. 5 resulted in highest pier-strength. Whether this strength could be maintained in masonry subjected to the ordinary wall exposures is another matter for consideration later. It should be emphasized

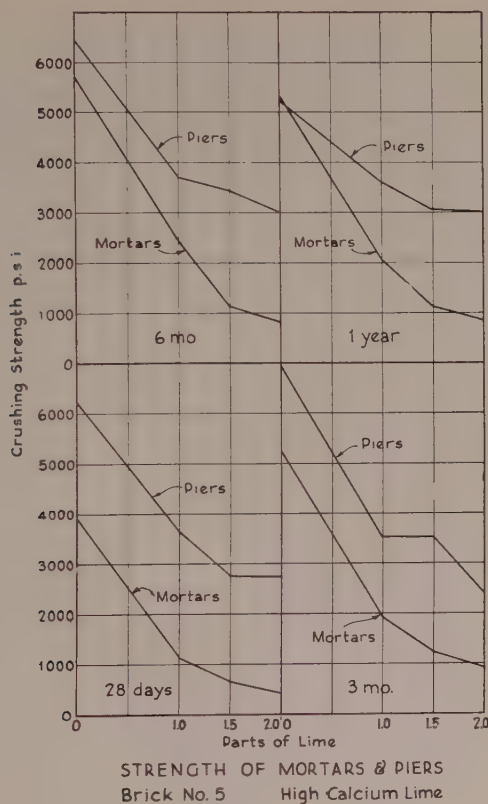
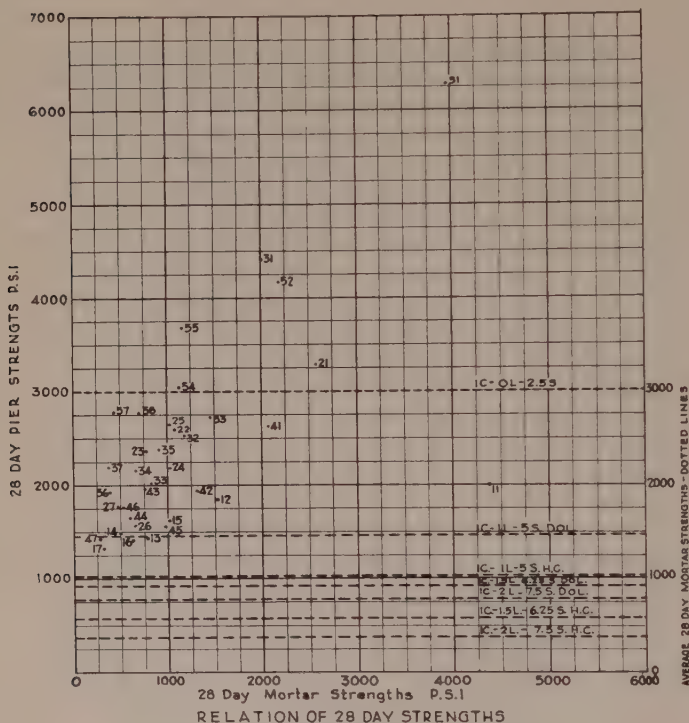


FIG. 5

here that, ordinarily, structural failure of masonry in use is a failure in detail and these failures do not occur due to concentric compressive loads, but may be accentuated by bending due to eccentric loading, depending upon the characteristics of the assembly loaded.

In Fig. 6 the 28-day pier-strengths are plotted against the 28-day strengths of the mortar cylinders. The average values for the five mortars made for use with each brick are shown by the dotted lines. Mortars must be made of variable consistencies depending upon various factors. The type of brick to be laid, the weather, the working characteristics of the mortar, and the idiosyncracies of the mason, all are factors that determine the consistency of mortar needed. Con-



sequently, where the mason is wetting mortars to his own liking for use with different brick under varying weather or working conditions, considerable variations in mortar strengths are to be expected. The most efficient and economical use of masonry demands a comprehension of the above facts as well as the fact that masonry-strength and mortar-strength are not synonymous terms. This latter point is well illustrated in the chart shown and, while there is no obvious exact trend shown, the general tendency for increased strength of pier with increasing mortar and brick strengths is clearly indicated. For purposes of comparison the ratios of pier-strengths to mortar-strengths and to brick strengths are listed in Table 1. It will be noted that the ratios of pier-strengths to mortar-cylinder strengths are quite generally higher for the mortars of lower compressive strength. This

NO.	MORTAR PROPORTIONS			LIME FLOW %	BRICK	PIER STRENGTHS				CYLINDER STRENGTHS				YOUNG'S MODULUS EX 10 ⁴	RATIO PIER TO CYLINDER	RATIO PIER TO REDUCED BRICK COMP.	PIER STL. REDUCED COMP. %
	C	L	S			28 Days	3 Mo.	6 Mo.	1 Yr.	28 Days	3 Mo.	6 Mo.	1 Yr.				
11	1-0-2.5	94	NONE		No. 1 SAND-STUCK 4840 Comp. Str. 709 Mod. Str. 167% Abr. Abn.	1997	2250	2555	2095	4385	5010	4860	5010	4.71	0.46	0.41	1335
12	1-1-5	60	DOL			1840	2380	2375	2276	1525	2057	2380	2295	3.71	1.21	0.38	1230
13	1-1.5-6.25	64	"			1425	1695	2345	1977	782	1449	1765	1477	3.81	1.82	0.29	955
14	1-2-7.5	68	"			1498	1720	1998	1720	511	1210	1525	1332	3.80	2.94	0.31	1000
15	1-1-5	62	H.C.			1607	1820	2178	1965	1015	1450	1840	1670	4.45	1.58	0.33	1075
16	1-1.5-6.25	57	"			1405	1395	2024	1623	629	1066	1390	1067	4.04	2.23	0.29	940
17	1-2-7.5	58	"			1324	1585	1748	1552	334	660	900	613	3.85	3.96	0.27	885
21	1-0-2.5	90	NONE		No. 2 SAND-STUCK 7900 1425% 14.5%	3280	4065	4000	4530	2583	4730	5870	5000	15.15	1.27	0.41	2195
22	1-1-5	75	DOL			2570	3235	3480	2935	1079	2220	2890	2717	8.08	2.38	0.33	1720
23	1-1.5-6.25	75	"			2360	2550	3013	2670	777	1960	2520	1867	6.38	3.05	0.30	1580
24	1-2-7.5	62	"			2180	2525	3028	2743	1026	1920	2410	1960	5.68	2.12	0.28	1460
25	1-1-5	61	H.C.			2640	3745	3020	3140	1024	1665	1805	1650	10.18	2.58	0.33	1770
26	1-1.5-6.25	36	"			2065	2870	3200	2580	647	1020	1280	1010	5.73	3.20	0.26	1380
27	1-2-7.5	56	"			1760	2465	3100	2440	477	709	900	706	3.59	3.69	0.22	1180
31	1-0-2.5	80	NONE		No. 3 WATER-STUCK 10400 1586% 10.1%	4400	4710	5080	4255	2007	3436	4890	3823	19.24	2.19	0.42	2945
32	1-1-5	67	DOL			2530	2710	3640	3260	1181	2230	2640	2617	5.97	2.14	0.24	1695
33	1-1.5-6.25	72	"			2015	2320	2850	2210	833	1350	2200	1753	5.53	2.42	0.19	1350
34	1-2-7.5	53	"			2160	2630	2770	2700	667	1280	1700	1400	5.55	3.24	0.21	1445
35	1-1-5	61	H.C.			2380	2935	3175	3115	911	1280	1765	1230	7.10	2.61	0.23	1595
36	1-1.5-6.25	53	"			1925	2355	2810	2380	520	955	1220	976	2.64	4.94	0.19	1290
37	1-2-7.5	57	"			2195	2775	2598	2620	370	775	860	843	1.82	5.94	0.21	1470
41	1-0-2.5	89	NONE		No. 4 SAND-LIME 4660 586% 10.1%	2620	2635	3015	2590	2067	4896	5110	4790	15.30	1.27	0.56	1755
42	1-1-5	73	DOL			1930	2345	2740	2440	1310	2166	2620	3400	6.91	1.47	0.41	1290
43	1-1.5-6.25	57	"			1760	1990	2028	2035	764	1586	1870	1472	6.10	2.30	0.38	1175
44	1-2-7.5	62	"			1650	1915	2140	1935	600	1560	2040	1520	4.65	2.75	0.35	1105
45	1-1-5	63	H.C.			1590	2155	2238	1740	984	1610	2120	1540	4.09	1.57	0.33	1035
46	1-1.5-6.25	57	"			1765	1720	2063	1780	525	1088	1395	1025	3.76	3.36	0.30	1180
47	1-2-7.5	62	"			1425	1575	1640	1445	282	610	759	505	2.66	5.04	0.31	955
51	1-0-2.5	79	NONE		No. 5 STIFF-MUD CLAY 9530 1758% 15%	6260	6965	6455	5290	3953	5240	5720	5388	26.80	1.58	0.65	4200
52	1-1-5	70	DOL			4155	4785	4800	4450	2184	3290	2730	3300	14.10	1.90	0.44	2785
53	1-1.5-6.25	55	"			2710	3745	3870	3515	1451	2270	2080	2200	10.50	1.87	0.28	1815
54	1-2-7.5	50	"			3040	3450	3625	3440	1124	2230	1830	1860	7.73	2.70	0.32	2035
55	1-1-5	74	H.C.			3670	3530	3725	3610	1165	1910	2440	2080	9.10	3.15	0.39	2460
56	1-1.5-6.25	55	"			2760	3515	3447	3095	692	1260	1145	1125	5.19	4.00	0.29	1850
57	1-2-7.5	62	"			2770	2420	3015	3025	437	926	813	870	4.36	6.33	0.29	1855

** Not taken

* At 50% Ultimate Load

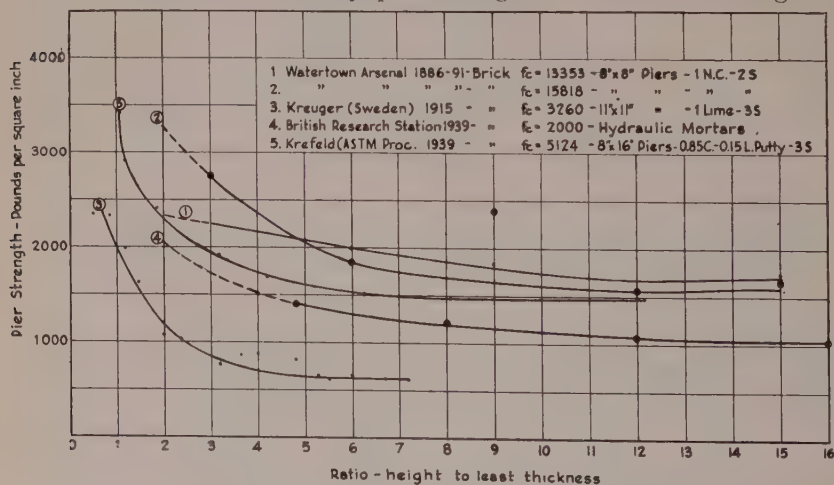
° Pier Strength x 0.67

TABLE 1—PIER AND CYLINDER STRENGTHS

is easily accounted for by the fact that the lower-strength mortars used in this series have a greater plasticity which results in a more complete bedding due to the intimate and continuous contact of mortar with the brick, thereby enabling the resulting masonry to take a proportionately greater load. This latter fact is very important from a long-time consideration of structural integrity, for it is a well-established fact that the use of mortars which have inherent plasticity and can come into intimate and continuous contact with the unit when being laid, result in masonry which suffers the least structural impairment in use. It is known that a very fluid and rich portland-cement

mortar or grout, when used with a porous unit, will result in a bond between the brick and mortar of tremendous 28-day strength, because of the pseudo-plasticity imparted by the excess water in the mortar, thereby permitting a more intimate and continuous contact between the mortar and the unit. That is, the set of the cement or at least the attainment of rigidity is somewhat delayed, and the mortar remains in a "plastic" state for a longer period of time due to the large excess of water, thus enabling a more intimate contact. It is also known that if such an assemblage is allowed to lie in a laboratory rack for a few years the strength of bond deteriorates to a very small value due merely to volume change of the mortar. The unfavorable characteristics of such a combination legislate against its use in masonry. In the case of the portland-cement mortar, the less fluid the mortar and the higher the absorption of the unit, the less intimate the contact initially. The ratios for 28-day pier strength to mortar-cylinder strength vary from 0.46 for the portland-cement mortar and Brick No. 1 to 6.33 for the 1C-2L-7.5S mortar and Brick No. 5. Thus it is seen that the requirement of a certain strength of mortar in a cube or cylinder does not give a true indication of the value of the mortar in the masonry.

The ratios for the 28-day pier-strength to the brick strength in



RELATION OF STRENGTH & SLENDERNESS RATIO

FIG. 7

compression vary from a low of 0.19 to a high value of 0.65 and these ratios are highest for the portland cement mortar as would be expected.

EFFECT OF SLENDERNESS RATIO

The strength of masonry piers decreases with increase in the ratio of height to thickness (h/t) and, since these tests were made upon specimens having an h/t ratio of 2, corrections must be made for a greater slenderness ratio of actual walls. In order to show how this factor influences the strength of piers, data from the various sources indicated were plotted. These curves are shown in Fig. 7. The tabulation below lists the reduction-factors read from these curves.

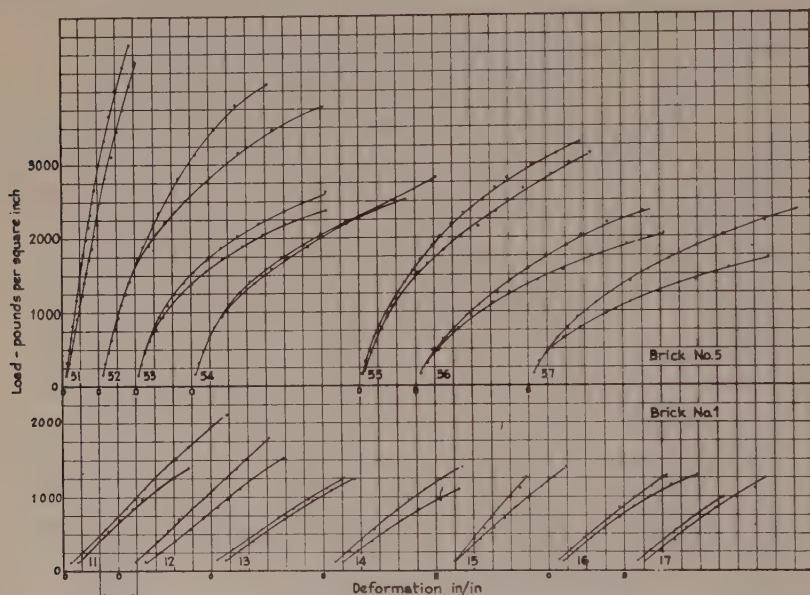
Test made by	Change in h/t	Reduction factor
Watertown Arsenal, 1886	2 to 12	0.68*
Watertown Arsenal, 1891	2 to 12	0.44*
Kreuger, Sweden, 1915	2 to 7.5	0.50
British Res. Station, 1939	2 to 12	0.50*
Krefeld (Columbia), 1939	2 to 12	0.64

*By extrapolation

Though the tests have been made at various times and with different materials the curves are similar, though variable. As it was necessary to extrapolate in three instances, (shown by dotted portion of curves) these values obtained by extrapolation are not as significant as the un-extrapolated value given by Krefeld's data. Kreuger's data while not considered of significance here, is interesting from the standpoint of comparison alone. If the curve for Kreuger's data was extrapolated the value for the reduction factor would remain practically the same as shown.

The reduction-factor value to transfer to wall strengths recommended by Krefeld for use with a test specimen with $h/t = 2$ is 0.67 which is probably conservative for masonry subjected to end restraint and is probably more nearly correct for small piers in a wall. In any event it would appear that Krefeld's data more nearly approximates the correct values for use with these piers due to the fact that the materials used would more nearly approximate those used in these tests and the value can be obtained without extrapolation. Accord-

ingly, the reduced 28-day masonry strengths are shown in Table 1 using a reduction-factor of 0.67 for the 28-day pier-strengths.



TYPICAL LOAD-DEFORMATION CURVES

FIG. 8

ECCENTRIC LOADING

Where eccentric loading is encountered, as is usually the case with masonry bearing-walls, the methods of ordinary analysis for eccentric loads have been shown by test and measured deformations to be correct for masonry work.* While the stress obtained from deformations does not follow an exact straight-line relation, as determined by formula, the agreement is quite close. It is the writer's belief that with wall-bearing masonry of very high modulus of elasticity, the deflection of the floor system at time of loading tends to increase the eccentricity, due to the non-yielding character of the bearing. Until relieved by plastic flow in the masonry under the bearing, a greater bending stress than that calculated would occur in the wall and cracking may occur due to excessive tension at the outside face

*University of Illinois, Bulletin No. 27, Tests of Brick Columns, etc. Talbot and Abrams.

of the wall. In several instances known to the writer deflection of the concrete slab resting on a brick masonry wall has been a contributing factor to cracking of the wall. Where a masonry of lower modulus of elasticity is used, the plastic flow of the masonry under the bearing allows the masonry to adjust itself more quickly to the change in bearing stress due to deflection of the floor system. Thus the eccentricity increase would be much less, resulting in less bending in the wall. It is hoped that this assumption may be checked by photo-elastic methods at some future time.

MODULUS OF ELASTICITY

Values for Young's modulus were determined from the load-deformation diagrams, for which typical curves are shown in Figure 8. The secant modulus was taken at 50% of the ultimate load, using the average deformations of two opposite sides. The 28-day values are listed in Table 1. Each value listed is the average of two specimens. For all piers except those built with Brick No. 1 the modulus of elasticity is considerably higher for those built with portland-cement mortar than those with the lime-cement mortars. Variation with Brick No. 1 can be explained partly upon the basis of absorption of the brick. When a portland cement mortar is placed upon a brick having a rate and total amount of absorption as high as this brick the mortar almost at once attains a state of rigidity due to a large loss of water. This leaves the mortar in a more porous state than if it could retain the water and lose it slowly to the unit, meanwhile being densified by consolidation due to added wall load. Obviously, after hardening this mortar will not take as much load as a denser mortar of the same composition and it will deform more rapidly and a greater amount under load. The variations of the moduli with the compressive strength of the brick are plotted in Fig. 9. From this plot, it is clear that the same reasoning, followed in accounting for the lower strengths of piers using Brick No. 3, holds true for the lower moduli of these piers as compared to those built with Brick No. 5. The irregularities of Brick No. 3 are such that greater deformations are produced with smaller loads, due to the unequal pressure on the joints. The higher moduli for the sand-lime brick might be accounted for on the same basis, as these brick were much more regular in shape and size than the comparable clay brick.

PERCENTAGE LINEAR CHANGES

for $1\frac{1}{2}'' \times 1\frac{1}{2}'' \times 12''$ Mortar bars

C-L-S By volume*	No Lime		H.C. Putty		Dol. Putty		H.C. Hydrate		Dol. Hydrate	
	2-28 Days	28-168 Days	2-28 Days	28-168 Days	2-28 Days	28-168 Days	2-28 Days	28-168 Days	2-28 Days	28-168 Days
1-0-2.5	+0.021	-.116								
1-1-5			+0.015	-.139	+0.020	-.107	+0.009	-.115	+0.032	-.091
1-2-7.5			+0.017	-.164	+0.014	-.106	+0.005	-.107	+0.029	-.088

2-28 days Moist curing at 70°F-95% R.H.

28-168 days In constant temp. room at 80°F-50% R.H.

*The lime proportion given is on the basis of dry hydrate

TABLE 2

VOLUME CHANGES

Masonry materials, like concrete, swell when wetted or heated and shrink when cooled or dried. The movement due to moisture change is materially higher than that due to temperature change and the coefficients for brick are lower than for mortars. For a homogeneous material inconstancy of volume is not such a serious factor as for non-homogeneous material, like masonry. If one realizes the possibilities of differential change between the mortar and brick, as well as the result of such changes, then proper steps can be taken to choose a combination of materials which will produce the minimum change. Conditions are not always such that the fullest advantages can be taken of the possibilities and in that case there must be a compromise.

It is obvious that structural deterioration may occur if differential changes are great enough to cause a rupture of the bond between the brick and mortar. That such failures of bond do occur have been shown by a study of field specimens taken from old walls of various ages throughout the country.*

In Table 2 are given the percentage length changes of $1\frac{1}{2}'' \times 1\frac{1}{2}'' \times 12''$ bars, made of various mortars, during 28-day moist curing and a subsequent drying period of 140 days in a constant-temperature room. After curing, the mortars were subjected to the drying period, in order to let the bars come to a more or less equilibrium condition. The length changes are less for the lime-cement mortars than for the portland-cement mortar, except for those made with the high-calcium quick-lime putty.

*A. R. E. A. Bulletin 396. Petrographic Study of the Bond Between Brick and Mortar. H. R. Staley.

After the bars had reached an equilibrium condition they were subjected to wetting and drying cycles, being immersed in water for five days, then allowed to dry for thirty-three days and the cycle then repeated. These changes during ten such cycles are shown

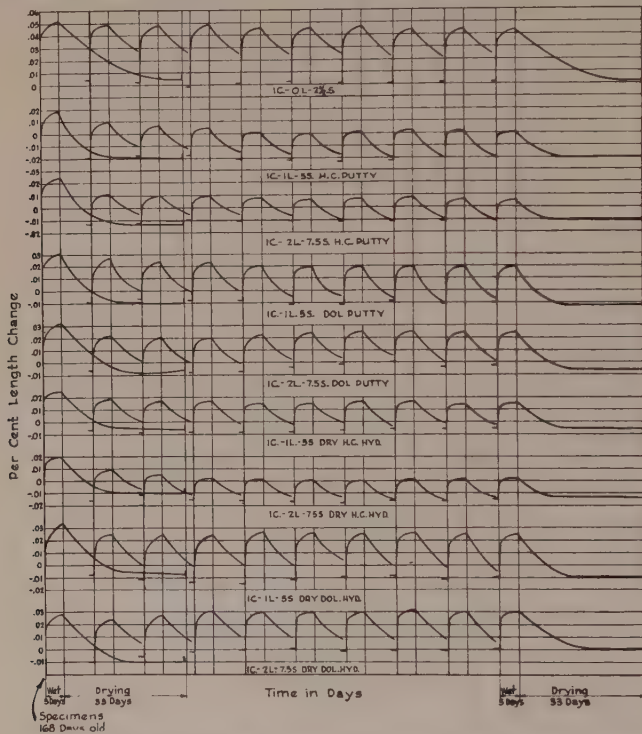


FIG. 10—LENGTH CHANGES DURING WETTING AND DRYING CYCLES

plotted in Fig. 10. To conserve space, the first and tenth cycle are plotted complete, while the others have only the wetting and a portion of the drying cycle shown. These changes are significant from a structural standpoint as it is this sort of cyclic change which has a bearing upon structural integrity. It is known that the changes measured on these unrestrained bars, cast in metal molds, are not absolute values for the same mortars in a wall, but the same relative values should hold even in the wall. It is this type of cyclic change that affects the permanency of the bond between the brick and the

mortar. The amount of volume change of the mortar and brick in the wall, and therefore the structural integrity, is a direct function of the penetrability of the masonry to moisture of all kinds. If moisture can be kept out, then a minimum of change occurs, as only temperature change will then be a factor. The portland-cement mortar has a greater length change in all cycles than the lime-cement mortars, the smallest cyclic changes occurring with the mortars containing the largest amount of high-calcium putty or hydrate. Thus, the greatest

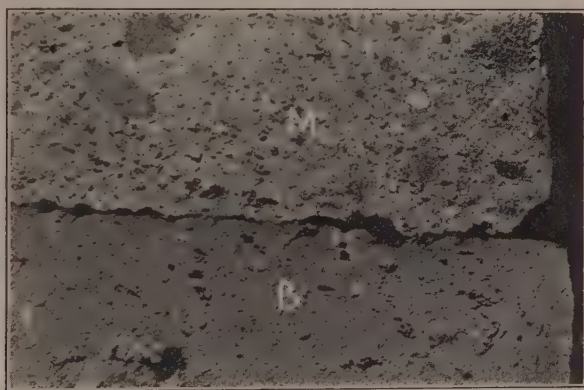


FIG. 11—MAGNIFICATION APPROXIMATELY $1\frac{1}{2}$ TIMES

cyclic change due to change in moisture occurs with the mortar which has the highest modulus of elasticity and produces masonry of the highest modulus. In addition the portland-cement mortar quite generally produces a tentacular or "spotty" type of bond with the brick which is conducive to penetration of water between the mortar and brick. The chances are good then for cyclic changes such as are shown to occur in the wall with this kind of mortar. These cyclic changes are one of the factors in causing rupture of the bond between the brick and mortar such as is shown in Figs. 11, 12, and 13. These photographs are taken of sections sawed from the brick-and-mortar specimens. The specimen shown in Fig. 11 was taken from a masonry wall nine years ago. The brick used was medium-high-absorption sandstruck laid up in a 1 portland cement-2 sand mortar. The wall leaked badly from the time it was built and it is very obvious where the leakage has occurred. There evidently was never a continuous

bond between mortar and brick and there has been a progressive loss of bond since the wall was built. This specimen came from a wall in New England.

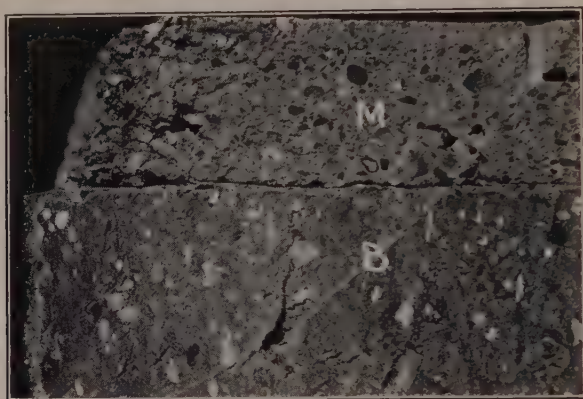


FIG. 12—MAGNIFICATION APPROXIMATELY $1\frac{1}{2}$ TIMES

Figs. 12 and 13 are of another brick-and-mortar specimen, at different sections. The loss of bond is perhaps more obvious in these photographs than in Fig. 11, though in all these sections it is visible to the naked eye. This particular wall was built of a shale, side-cut brick, laid in a mortar of 1 portland cement-3 sand, with 25% of lime. This wall did not leak until about five years after it was built



FIG. 13—MAGNIFICATION APPROXIMATELY $1\frac{1}{2}$ TIMES

and the wall was eleven years old when the specimen was taken. This specimen came from a wall in the middle-west and the conditions under which it was built are known personally to the writer.

Figs. 14 and 15 are also photographs of roughly sawed sections of a brick and mortar specimen. This specimen also came from a wall in New England and was thirty-seven years old when taken. The mortar was 1 cement-2 lime-7 sand with a water-struck brick. Note that the old mortar (M and O.M.) still has an intimate and continuous contact with the brick, while the rich, portland-cement pointing mortar (P.M.), even though placed later than the original mortar, has cracked loose from the brick. The characteristic be-



FIG. 14—MAGNIFICATION 20 TIMES

havior of these two mortars in use, shown in Fig. 15, makes it obvious that something besides high strength in the mortar and high strength in bond must be the criterion where structural integrity is of consequence. A more comprehensive study of the bond between brick and mortar has been published so it will not be discussed further here.*

EFFLORESCENCE

When water penetrates into a masonry wall some of the soluble salts present in the brick and/or mortar are carried to the surface of the wall where they are deposited when the water evaporates. All

*A. R. E. A. Bulletin 396. Petrographic Study of the Bond Between Brick and Mortar. H. R. Staley.

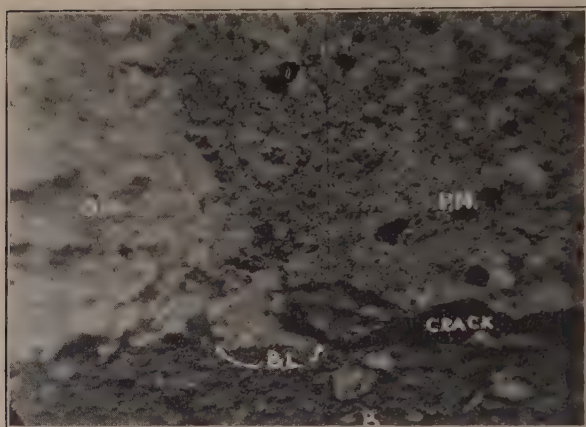


FIG. 15—MAGNIFICATION 20 TIMES

efflorescence is crystalline and severe disintegration may result from this crystal growth especially when it occurs beneath the surface of the mortar or brick. Hardesty** reported an example of such disintegration in 1939. Portland-cement mortars and products seem to be a more prolific source of soluble salts than the ordinary clay brick or lime-cement mortars.† Efflorescence promotes an insidious type of disintegration which is harmful, but generally slow in progress. Any type of disintegration or deterioration is harmful to structural stability and quite generally the conditions which promote deterioration in one manner are factors in all. For example, water penetration not only causes efflorescence, it also causes differential volume changes in the wall and water present in the wall at time of freezing temperatures can be very detrimental. Leakage of masonry and water penetration are the direct results of a poor choice of the combination of brick and mortar. The incompatibility of the brick and mortar are not the only cause for leaky masonry, but commonly this is the major factor in contributing to this condition. To make a correct choice involves a consideration of the characteristics of both the brick and the mortar and the manner in which they affect the assembly at the time of construction, as well as over a period of years. Details of wall-design,

**A. S. T. M. Proceedings, 1939. 'Disintegration of Face Brick by Crystallization of Soluble Salt. Hardesty.

†Technical Bulletin No. 3, Clay Products Technical Bureau, London, England.

improper use or lack of through-flashings, and workmanship are other contributory factors to leakage.

EFFECT OF TEMPERATURE AND MOISTURE CHANGES

Consideration should be given to some of the possible effects of temperature and moisture changes upon the finished wall on the basis of the information given in the tabulation below.

AVERAGE VALUES FOR COEFFICIENTS		
	Moisture Coefficient wet to dry Inches per inch	Temperature Coefficient Inches per inch deg. F
Masonry	.00012	.0000031
Concrete	.00040	.0000067
Portland-cement mortar	.00050	.0000093
Lime-cement mortar	.00025	.0000080
Clay brick	.00007	.0000050

As an example, consider the effects of a temperature change of 20° F. in a concrete slab 100 feet long, bearing on a masonry wall. This will produce a change in length of $100 \times 12 \times .0000067 \times 20 = 0.16"$. If this movement is transferred to a restrained masonry wall using a typical face brick such as No. 3 or No. 5, with a modulus of elasticity of say 20×10^5 , a stress of 267 pounds per square inch in the wall results. With a masonry wall having a modulus of elasticity of 4×10^5 the stress would be 54 pounds per square inch. The above values are based upon an assumed story-height of ten feet and a 12" solid wall. The strain in the ten foot height of wall would be one-tenth the movement of the slab. The stress in the masonry due to the movement of a concrete slab is a factor for serious consideration and quite often results in cracking of the wall at some point. In one instance, the expansion of a concrete slab cracked the entire length of the wall, inside, on a line with the tops of the windows. Other similar instances are rather common, especially where high-modulus masonry is employed. The tensile strength of bond of mortar to brick, as measured on laboratory specimens, seldom reaches a value of 150 pounds per square inch. If this limit is representative of field conditions, the significance of movement due to temperature and moisture changes becomes more apparent.

Movements due to temperature and moisture changes in the masonry wall itself are not so important unless the wall is restrained and of considerable length. The differential movement of the brick and mortar may be quite important and may result in local bond failure. Changes in moisture produce the larger movements and that masonry which is more permeable to water will be affected to a greater extent. Masonry built with mortars of high cement-content quite generally is more permeable to water and therefore would be more affected than less permeable masonry. This checks with field observations.

No attempt has been made to calculate the stress produced by differential changes between the brick and the mortar, as the exact change in actual masonry, is not definitely known. It is believed that the relative values shown in the tabulation for coefficients will hold even for masonry, so that we can expect a differential change in the ratio of 6-7 to 1 using a portland-cement mortar and 2-4 to 1 using lime-cement mortars, depending upon the proportion of lime used. Obviously the chances for maintenance of the bond between the brick and mortar favors the use of lime-cement mortars as shown in the photographs previously discussed.

CONCLUSION

An attempt has been made to show that there are other factors beside compressive strength, which should be considered when choosing a combination of brick and mortar for masonry and that for best results, from a standpoint of structural stability and integrity, there must be a compromise made in the selection. In my opinion, the best method of judging in the compromise is to consider the limitations of the assembly as well as its capabilities. To estimate the capabilities of an assembly of materials requires only an intelligent analysis of tests, but to estimate the limitations requires a judgment acquired by experience in using the materials and keen observation of their behavior in use.

In the estimation of the capabilities and limitations the following facts should be kept in mind.

In order to obtain some of the other desirable characteristics in masonry walls some sacrifice of strength can be made. The use of

adequate strength rather than highest strength is a more desirable procedure. Adequate strength of masonry can be obtained by using mortars with at least one part of lime to one part of portland cement. With the use of the stronger brick adequate strength can be obtained with mortars higher in lime content.

Structural integrity is a direct function of the intimacy and continuity of bond between the brick and mortar and the maintenance of this bond throughout the life of the wall. Lime-cement mortars possess the properties to fulfill the conditions for promoting structural integrity as the contact between the brick and mortar is more intimate at the time of laying and the tendency to maintain the original contact is greater with this type of mortar. Water penetration through masonry is also a function of the intimacy and continuity of bond.

The above facts have been verified by field usage and no stronger proof can be offered than is given by successful usage under field conditions.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

September 18, 1940.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the Engineers' Club and was called to order by Vice-President, Arthur L. Shaw. This meeting also was a joint meeting with the Society of American Military Engineers, Boston Post. One hundred and seventy members and guests were present; one hundred thirty-seven persons attended the dinner preceding the meeting.

Vice-President Shaw, presiding for the business meeting of BSCE, announced the death of the late President, Frank B. Walker, who had been a member since October 19, 1915, and who died June 3, 1940.

In consequence of Mr. Walker's death, an announcement was made that the Nominating Committee had submitted a nomination to fill the unexpired term of President. Vice-President Shaw requested Past President, Gordon M. Fair to act as Chairman *pro tem* to preside during the election. Prof. Fair read the report of the Nominating Committee, presenting the name of Arthur L. Shaw for President to fill the unexpired term. On motion made and seconded, VOTED that the report

of the Nominating Committee be accepted, that the nominations be closed and that the Secretary be directed to cast one ballot for the office of President.

The Secretary thereupon cast one ballot for President and Prof. Fair then declared that Arthur L. Shaw had been elected President. Mr. Shaw resumed the chair and expressed his appreciation of the confidence in him as indicated by his election to the office of President.

President Shaw announced that the Annual Student Night will be held on October 16, 1940, and the speaker will be Col. Charles R. Gow, President of Warren Bros. Inc.

President Shaw, concluding the items of business of BSCE, introduced Commander John P. Searles, President of the Society of Military Engineers, Boston Post, and turned the meeting over to him.

Commander Searles introduced the guests at the head table, including Rear Admiral W. T. Tarrant, Commandant, First Naval District; Col. Leonard B. Gallagher, District Engineer, U. S. Army, Boston District; Col. J. S. Bragdon, District Engineer, U. S. Army, Providence, R. I.; Major David B. Weden, Representing His Excellency, Governor Leverett D. Saltonstall; Den-

nis J. Delaney, WPA Administrator for Massachusetts; President, Arthur L. Shaw, of Boston Society of Civil Engineers; Holcombe J. Brown, President, Engineering Societies of New England.

Commander Searles then introduced the guest speaker, Major General Julian L. Schley, Chief of Engineers, Corps of Engineers, U. S. Army, who gave a particularly interesting talk on the work and functions of the U. S. Engineers.

At the conclusion of the address, the speaker was given a rising vote of thanks.

EVERETT N. HUTCHINS, *Secretary*.

October 16, 1940.—A regular meeting of the Boston Society of Civil Engineers was held this evening at Pritchett Hall, Walker Memorial, Massachusetts Institute of Technology, and was called to order by President Arthur L. Shaw at 7:15 o'clock. Two hundred forty members and guests were present and 204 persons attended the dinner.

This meeting was the Annual Student Night with representations from the Student Chapters of the American Society of Civil Engineers at Harvard University, Massachusetts Institute of Technology, Tufts, Rhode Island State College, New Hampshire State University, Brown University, Worcester Polytechnic Institute and of Northeastern University, which also is a Section of the BSCE. This meeting was a joint meeting with the Northeastern Section, American Society of Civil Engineers.

The President extended a welcome to the 160 students who attended this meeting and expressed his appreciation of the coöperation of the officers of the student organizations and of the faculty members in making this event successful.

President Shaw then introduced the speaker of the evening, Col. Charles R. Gow, a Past President of the So-

ciety and President of Warren Brothers Corp., who gave an address on "Engineering Personality as an Aid to Professional Success."

The meeting adjourned at 9:00 P.M.
EVERETT N. HUTCHINS, *Secretary*.

APPLICATIONS FOR MEMBERSHIP

[October 20, 1940]

The By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

For Admission

EARL R. BAKER, Wollaston, Mass. (b. January 1, 1889, Green, Kansas). Graduated from University of Vermont, 1912, B.S. degree. Experience, May, 1912 to August, 1912, Draftsman, The George A. Fuller Company, Montreal, Canada. August, 1912 to November, 1912, Draftsman, National Bridge Company, Montreal, Canada. November, 1912 to September, 1914, Designer and Draftsman, Phoenix Bridge & Iron.

Works Ltd., Montreal, Canada. September, 1914 to March, 1916, Computer, Vermont Experiment Station, Burlington, Vermont. March, 1916 to May, 1917, Stone & Webster, Boston, Mass., Designer and Checker. May, 1917 to August, 1918, Designer and Estimator, Ferguson Steel & Iron Company, Buffalo, New York. August, 1918 to September, 1919, Chief Draftsman and Chief Engineer, Republic Carbon Company, Niagara Falls, New York. June, 1921 to November, 1921, general estimating, Lupfer & Remick, Buffalo, New York. November, 1921 to May, 1922, Estimator and Salesman, R. S. McMannus Steel Construction Co., Buffalo, New York. May, 1922 to July, 1925, Estimator and Salesman, The A. L. Smith Iron Works, Chelsea, Mass. July, 1925 to November, 1925, Salesman, Standard Steel & Iron Works, Watertown, Mass. November, 1925 to March, 1932, Sales Engineer, Palmer Steel Company, Boston, Mass., and Portland, Maine. April, 1932 to December, 1936, in charge of sales, Ralph Coal Company, Wollaston, Mass. December, 1936 to December, 1937, Structural Designer and Checker, Stone & Webster, Boston, Mass. December, 1937 to December, 1938, Structural Designer and Checker, Jackson & Moreland, Boston, Mass. October, 1938 to October, 1939, Structural Designer, E. B. Badger & Sons Co., Boston, Mass. At present with Metropolitan District Water Supply Commission, Boston, Mass., as Assistant Civil Engineer. Refers to *C. L. Coburn, G. Coffin, H. G. Dresser, H. Gray, O. G. Julian.*

FRANK L. CHENEY, Needham, Mass. (b. April 1, 1906, Westwood, Mass.). Northeastern University, 1923 to 1927 incl. Experience, Instrumentman, Connecticut State Highway Department in 1928. Assistant Engineer with W. Worthington, Engineer, 1928 to March, 1930. Assistant Town Engineer, Town

of Needham, Mass., March, 1930 to April, 1934. Town Engineer, Needham, Mass., April, 1934 to date. Supt. Engineering Division of Public Works Dept., Needham, Mass. Inspector of Sand and Gravel Pits. Refers to *P. E. Bodemer, F. L. Flood, C. B. Humphrey, F. H. Kingsbury, G. H. Meserve, Jr., H. L. White.*

LANGLEY STODDARD HOMER, Newton, Mass. (b. September 6, 1890, Brooklyn, N. Y.). Graduated from Rensselaer Polytechnic Institute, 1913, degree C.E. Experience, 1914-1917, Asst. Engineer with Wm. J. Wilgus, Consulting Engineer of New York City on Railroad valuation work. 1917-1918 and part of 1919, 1st Lieut. and Captain, Corps of Engineers, U. S. Army as assistant to Col. Wilgus, Deputy Director General A. E. F. and also in Command of Company 304th Engineers—79th Combat Division. 1920-1940 and at present Superintendent of Construction, Turner Construction Company in charge of construction of over Thirty Million Dollars of Buildings. Licensed as Professional Engineer, State of New York. Member of American Society of Civil Engineers. Lecturer at Harvard Graduate School of Engineering. Refers to *A. Casagrande, M. N. Clair, H. A. Mohr, S. Thompson.*

MARTIN ALEXANDER MASON, Chevy Chase, Md. (b. April 23, 1907, Washington, D. C.). Attended George Washington University evening classes receiving B.S. degree in Engineering in 1931. Experience, Employed in currency paper research, and testing of instruments at the National Bureau of Standards. Placed in charge of current meter rating station at National Bureau of Standards, January, 1929, and continued graduate study at Johns Hopkins University, National Bureau of Standards Graduate School, and George Washington University. Re-

ceived Freeman Scholarship from Boston Society of Civil Engineers, July, 1937, and studied in European hydraulic laboratories for 15 months. Received degree of Doctor of Engineering from University of Grenoble, Grenoble, France. Returned to National Hydraulic Laboratory November, 1938, worked on water measurement methods, water hammer and hydraulic model testing. Appointed Chief Research Section of Beach Erosion Board, War Department, in April, 1940. Refers to *C. M. Allen, L. J. Hooper, E. N. Hutchins.*

GORDON SCOTT RUTHERFORD, Wellesley Hills, Mass. (b. October 19, 1875, Montreal, Canada). (Naturalized Citizen 1904). Graduated from McGill University, M.R., 1896. Post Graduate of McGill University, 1897. Experience, Assistant plant manager, Solvay Process Company, at Syracuse, Detroit, Hutchinson, Kansas, Ampersburg, Ontario, in charge of operations, construction and maintenance at their several plants while resident manager. 1925 to 1938, with Diamond Alkali Company, in charge of Chemical operation, construction, etc., plant at Painesville, Ohio. At present with New England Concrete Pipe Corp., Newton Upper Falls, Mass., duties, executive and Consulting Engineer. Refers to *M. N. Clair, S. E. Thompson.*

P. CHARLES STEIN, Cambridge, Mass. (b. February 12, 1905, Baltimore, Md.). Attended Baltimore Polytechnic Institute. Received C.E. Degree at Cornell University, June, 1925. Experience, 1926-1936 with Whitman Reguardt & Smith, Construction Engineers, Baltimore, Md., nature of work—surveys, reports, designs and specifications for water works, sewage systems, structures, etc. Awarded M.S., University of Iowa, June, 1937. Awarded Sc.D., Mass. Institute of Technology, June,

1940. At present instructor in Sanitary Engineering, Massachusetts Institute of Technology. Refers to *J. B. Babcock, C. B. Breed, T. R. Camp, Wm. D. Henderson, D. W. Taylor.*

Transfer from Grade of Junior

WILLIAM ROBERT WILLIAMS, Brookline, Mass. (b. April 1, 1912, Howe, Texas). Attended Texas Technological College, Sept. 1933-June, 1934; California Institute of Technology, Sept. 1935-June, 1936; Massachusetts Institute of Technology June, 1936-June, 1938. Received B.S. degree in Civil Engineering June, 1938, at Massachusetts Institute of Technology. Experience:—draftsman with D. A. Schulte Inc., June, July, 1938 and worked as a draftsman for M. P. Scullin, American Oil Products Company, July and August of 1938. From Sept., 1938 to present time employed as Junior Engineer by Water Resources Branch of U. S. Geological Survey, Boston. Refers to *J. B. Babcock, H. K. Barrows, H. B. Kinnison, C. M. Spofford.*

Transfer from Grade of Student

HARRY L. FREEMAN, Cambridge, Mass. (b. June 1, 1915, Boston, Mass.). Graduated from Northeastern University, June, 1939, with B.S. degree in Civil Engineering. Experience, Co-operative work periods, September to November, 1937, with Whitman & Howard, Boston, as rodman and transitman with land survey parties. September to October, 1938, with Carver Engineering Company, Boston, Mass., as rodman on survey of condition of piles of wharf. January to February, 1939, with W. A. Mason & Son Company, Cambridge, as rodman on preliminary survey of site for federal housing project. March to September, 1939, with Walter Channing, Inc., Boston, Mass., as surveyman and draftsman on surveys to determine damage done to private properties by hurricane and Metropolitan

aqueduct. September, 1939 to present, with U. S. Engineer Office, Providence, R. I., as assistant structural draftsman on fortifications. Refers to *C. O. Baird*, *C. S. Ell*, *A. E. Everett*, *E. A. Grams-torf*.

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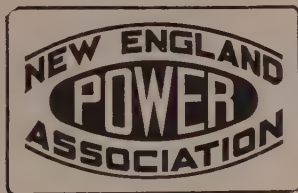
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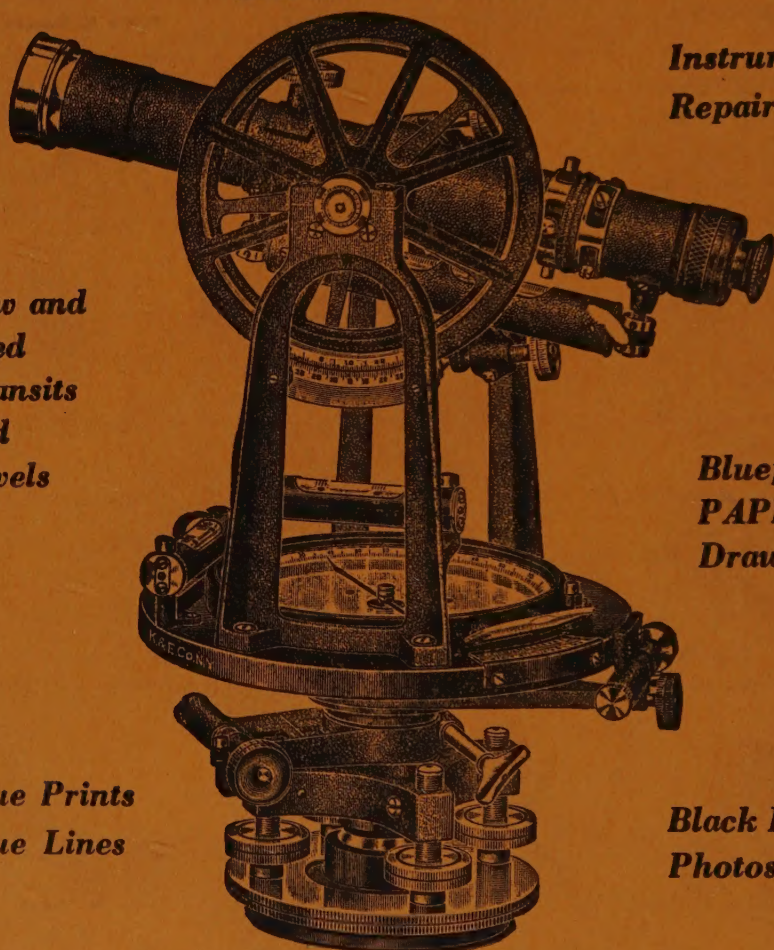
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